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THE STRENGTH OF MONOLITHIC CONCRETE WALLS

A REPORT OF AN INVESTIGATION

CONDUCTED BY

THE ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS

IN COÖPERATION WITH

THE PORTLAND CEMENT ASSOCIATION

BY

FRANK E. RICHART

AND

NATHAN M. NEWMARK

BULLETIN No. 277
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THE STRENGTH OF MONOLITHIC CONCRETE WALLS

I. Introduction

1. *Object and Scope of Investigation.*—The tests reported herein were made for the purpose of obtaining information on the strength and stability of monolithic concrete walls of types used in concrete house construction. The investigation was of the nature of a reconnaisance of the field rather than a detailed study. The functions of a wall for a residence may be divided into three classifications: (1) To provide safety; that is, to resist vertical loads, wind pressures, and other incidental forces, and to provide resistance to moisture and weathering as well as to fire. (2) To provide comfort; that is, to furnish heat and sound insulation, and to prevent percolation and condensation of moisture. (3) To be convenient and economical of construction and readily adaptable to architectural requirements. The fact that this report is concerned only with the strength and stability of concrete walls does not therefore mean that these are considered to be the only important factors.

There is very little published information available concerning the strength of monolithic concrete walls. Tests have been made on concrete block pilasters, and on the stability and strength of walls of concrete masonry units.*

There are reported herein results of tests on panels of single and double walls of various thickness, made of dry-tamped concrete and constructed in successive courses. Tests were also made on ribbed walls consisting of a slab and vertical ribs constructed as a single unit, and on single walls of the same size and shape as the dry-tamped single walls, all made of poured concrete. Eighteen large and six small wall panels and two ribbed walls were tested with a uniformly-applied axial vertical load. Two ribbed walls were tested with an eccentric load, and seven walls were tested in flexure.

2. *Acknowledgment.*—The investigation described herein was carried on at the Materials Testing Laboratory of the University of

*J. R. Shank and H. D. Foster, "Strength of Concrete Block Pilasters under Varied Eccentric Loading," Bulletin No. 60, Eng. Exp. Sta., Ohio State University, 1931.

C. A. Menzel, "Tests of the Fire Resistance and Stability of Walls of Concrete Masonry Units," Proc. A.S.T.M., Vol. 31, Part II, p. 607, 1931.

F. E. Richart, P. M. Woodworth, and R. B. B. Moorman, "Tests of the Stability of Concrete Masonry Walls," Proc. A.S.T.M., Vol. 31, Part II, p. 687, 1931.

F. E. Richart, "The Structural Performance of Concrete Masonry Walls," Jour. Am. Conc. Inst., Vol. 3, No. 6, p. 363, February, 1932.

F. E. Richart, R. B. B. Moorman, and P. M. Woodworth, "Strength and Stability of Concrete Masonry Walls," Bulletin No. 251, Eng. Exp. Sta., University of Illinois, 1932.

Illinois, in coöperation with the Portland Cement Association, Chicago, Illinois. The work was done as a regular project of the Engineering Experiment Station, under the administrative direction of Dean M. L. Enger, Director of the Station, and Prof. F. B. Seely, Head of the Department of Theoretical and Applied Mechanics. The Portland Cement Association was represented by an Advisory Committee consisting of Mr. Frank T. Sheets, Consulting Engineer, and Mr. R. E. Copeland.

The Arro-Tie Company of Kansas City, Missouri, furnished the wall ties used in the construction of the ribbed walls and the solid poured walls.

II. Construction and Testing of Walls

3. *General Description of Walls.*—It was the primary purpose of these tests to furnish information on the behavior under load of wall panels of sufficient size to represent story-height building construction. Therefore, wall panels six feet long and nine feet high were chosen. Several small wall panels 32 in. long and 48 in. high were used also to determine whether or not they could be established as representative test specimens.

To provide a rigid base on which the walls could be built, transported, and tested, reinforced concrete pallets were used. These were 5¾ in. thick, of sufficient area to support the wall, and were poured on a plane machined surface so that no bedding was required when pallet and wall were placed in the testing machine. The pallet was stiff enough to prevent appreciable deflection during the fabrication of the wall. Preparatory to building a wall the pallets were placed on two carefully-leveled steel rails and the frame work and guides for the sliding forms were built up on the pallets and carefully plumbed. This insured that the completed wall would be plumb and normal to the plane of the base. The walls were anchored to the pallets by ½ in. steel dowel bars 15 in. long, spaced about one foot apart.

Three different mixes of dry-tamped concrete and two mixes of poured concrete were used in the construction of the walls. The details of the mixes and the properties of the aggregates are given in Table 1. The dry-tamped mixes, A, B, and C, were designed to give medium, high, and low strength concretes. Mix D was designed to give a medium-strength concrete of sufficient workability to be placed with only a normal amount of spading and no mechanical vibration in the thin reinforced ribbed walls. Mix E was designed to represent an

TABLE 1

DETAILS OF CONCRETE MIXES USED IN WALLS

Mix	Proportions by Dry, Loose Volume				Proportions by Weight				Remarks
	Cement	Sand	Gravel	Water	Cement	Sand	Gravel	Water	
A	1	2.50	4.50	0.73	1	2.70	4.93	0.485	Dry tamped
B	1	1.50	2.75	0.52	1	1.62	3.01	0.345	Dry tamped
C	1	4.00	6.00	1.00	1	4.31	6.57	0.664	Dry tamped
D	1	2.50	3.50	1.05	1	2.70	3.83	0.697	Slump = 8 in.
E	1	2.75	3.88	1.00	1	2.96	4.25	0.664	Slump = 5-6 in.

Gravel for mix D screened through ¾-in. screen.
Unit weight by dry loose volume:
sand.....101.3 lb. per cu. ft.
0-1½ in. gravel.....103.0 lb. per cu. ft.
0-¾ in. gravel.....102.6 lb. per cu. ft.

Sieve analysis of aggregate

	Per cent passing sieve									
	No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜-in.	¾-in.	1-in.	1½-in.
Sand.....	1	7	50	77	87	97	100	....	....	...
Gravel.....	...	...	...	...	1	8	52	87	99	100

easily-workable concrete of the same strength as the medium-strength dry-tamped concrete.

An outline of the make-up of the various walls and the tests made on them is given in Table 2. Four groups of four-inch single walls were made, three of dry-tamped concrete of different strengths, and one of poured concrete. Four groups of double walls of various thickness were made, all of the same grade of concrete. Six-inch single walls, ribbed walls, and three groups of wallettes, or small walls, complete the outline.

A drawing of typical single and double walls and of a ribbed wall showing the details of the placing of reinforcement is shown in Fig. 1.

4. *Construction of Walls of Dry-Tamped Concrete.*—The walls of dry-tamped concrete were constructed in a manner similar to that used in the field construction of such walls, in courses approximately 9 in. in height. Successive courses were placed at intervals of about two hours. Side forms were removed shortly after the placing of the course and the concrete was left exposed to dry and harden so that the next course could be placed. The forms consisted essentially of sliding channel-beams guided by vertical posts. The posts were anchored to the pallets and tied together and braced by wood struts.

Table 2

Outline of Tests of Monolithic Concrete Walls

Large walls except ribbed walls—6 ft. long, 9 ft. high.
Small walls—2 ft. 8 in. long, 4 ft. high.
Ribbed walls—2 in. slab, 5 ft. 6 in. long, 9 ft. high; 2 ribs 3 in. wide and 4 in. deep, spaced 33 in. o.c.
All walls except Nos. 8 and 10 made of dry-tamped concrete constructed in vertical lifts of 9 in.
Horizontal reinforcement—¼-in. round bars, 9 in. o.c.
Ties—¼-in. round bars, spaced 9 in. vertically and 12 in. horizontally.
Reinforcement in ribbed walls—one ½-in. round bar in each rib; wire mesh, No. 6 wires 4 in. o.c. horizontally and vertically, in slab.

Group No.	Type of Wall	Thickness, inches				Concrete Mix*	Reinforcement	Number of Walls Tested	
		Total	Inside Wall	Outside Wall	Space			Compression	Flexure
1	Large solid wall	4	....	....	....	A	Horizontal	2	1
W1	Small solid wall	4	....	....	....	A	Horizontal	2	...
2	Large solid wall	4	....	....	....	B	Horizontal	2	...
3	Large solid wall	6	....	....	....	A	Horizontal	2	...
4	Large double wall	10½	4	4	2½	A	Horizontal and Ties	2	1
W4	Small double wall	10½	4	4	2½	A	Horizontal and Ties	2	...
5	Large double wall	8½	3	3	2½	A	Horizontal and Ties	2	...
6	Large double wall	12½	6	4	2½	A	Horizontal and Ties	2	...
W6	Small double wall	12½	6	4	2½	A	Horizontal and Ties	2	...
7	Large double wall	9½	4	3	2½	A	Horizontal and Ties	2	...
8	Ribbed wall	6	....	....	....	D	Vertical and Mesh	4†	3‡
9	Large solid wall	4	....	....	....	C	Horizontal	2	1
10	Large solid wall	4	....	....	....	E	Horizontal	2	1

*See Table 1.
†Two walls loaded axially, two eccentrically.
‡Two tests were of walls that had previously failed under eccentric loading.

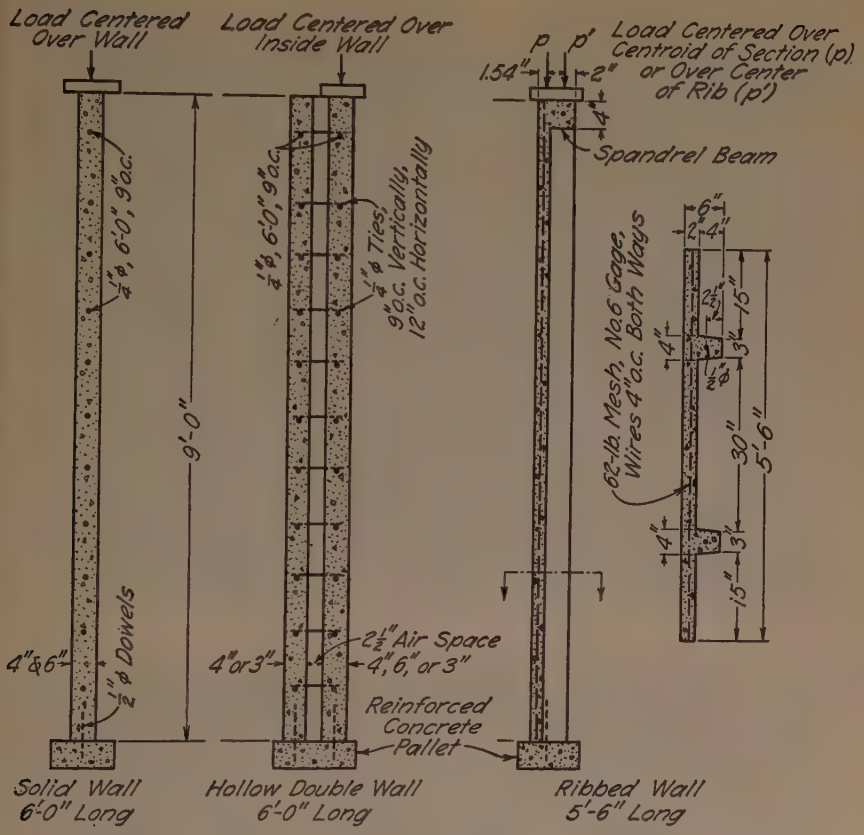


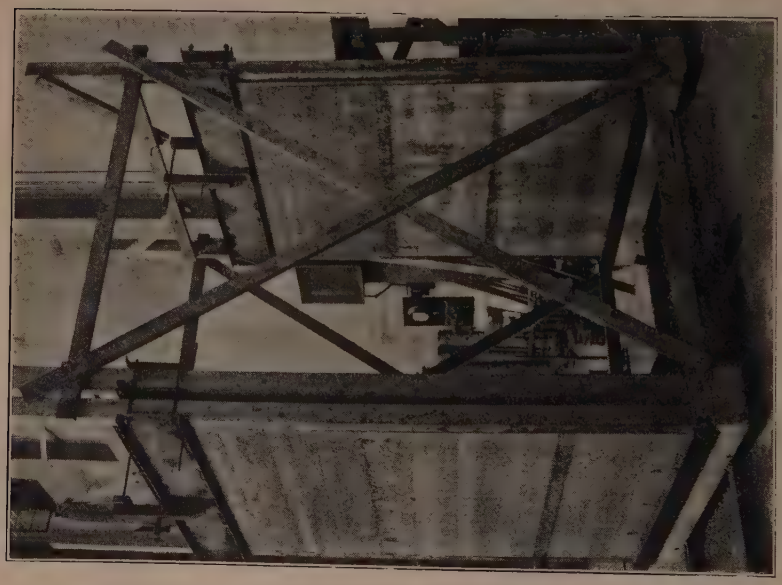
FIG. 1. DETAILS OF TEST WALLS

The side forms or sliding forms were steel channels 12 in. deep made from 12-in. J. and L. Junior I Beams by milling off the flange on one side. A view of the forms for the large walls is shown in Fig. 2. It will be noted that forms were set up for two walls at a time, and that the walls were braced to each other.

The forms for the hollow walls were similar in appearance to those for the solid walls. Two posts at each end of the wall were used as guides for the sliding forms. The forms used for the outer faces of the hollow walls were the same as those used for the single walls. The forms for the inside faces were made of notched boards with the notches cut in such a way that the boards fitted down over the steel ties which held the two parts of the wall together.

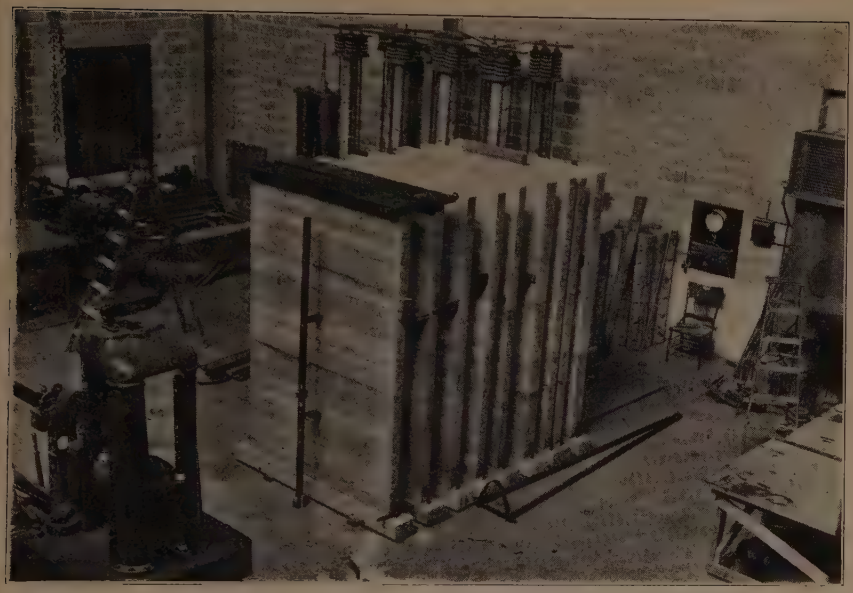
A view of the forms during the construction of different walls is

(b)

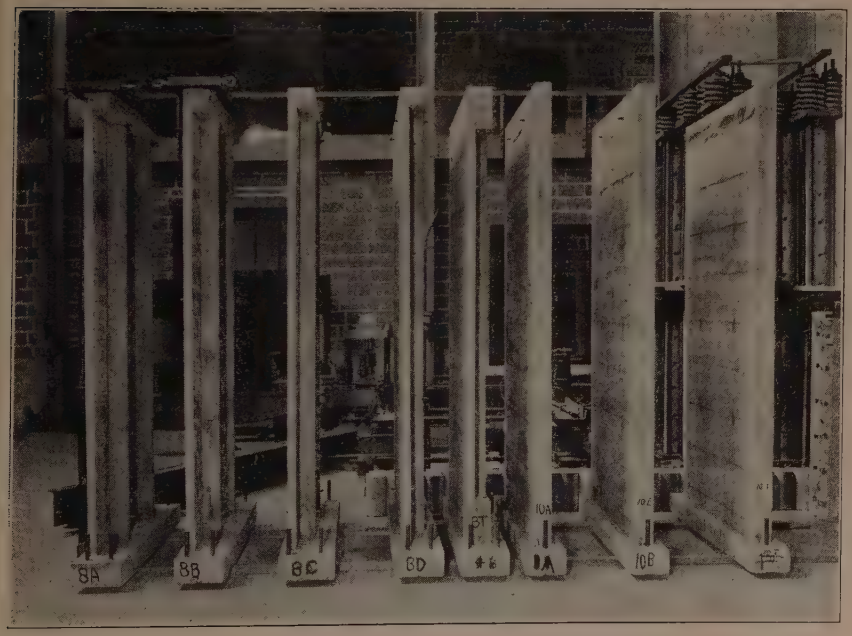


(a)

FIG. 2. CONSTRUCTION OF (a) DRY-TAMPED WALLS AND (b) RIBBED WALLS



(a)



(b)

FIG. 3. DRY-TAMPED (a) AND RIBBED (b) WALLS IN STORAGE

shown in Fig. 2(a). In the foreground is shown a double 4-in. wall, and in the background a 6-in. solid wall.

The concrete in the walls was tamped in two layers for each course with rectangular tamps about 2 to 2½ in. by 7 in., weighing approximately 5 lb. The tamping was carried on until the concrete became compacted and appeared to be slightly mushy. This ordinarily took about 5 minutes per course in the 4-in. single walls.

In general, walls of different groups were made at the same time so that duplicate specimens of the same group were completed on different dates.

Control cylinders were made with each wall, usually with about every other course constructed. The concrete was placed in the control cylinders in 4 in. layers and tamped with a round wooden or metal tamp until the concrete became slightly mushy. This averaged about twenty strokes for each layer of concrete. The tamp used was 2¼ in. in diameter at the end and weighed about 4 lb. The amount of tamping given to the cylinders was the same as that given to the walls, as closely as could be estimated. The cylinders were capped with neat cement a few hours after they were made and the molds were removed within 12 to 24 hours. Three control cylinders were made at a time, one of which was tested at 7 days and the other two at 28 days. The 7-day cylinders and half of the 28-day group were stored in the laboratory with the walls; the remainder were stored in the moist room as soon as the molds were removed.

The walls were stored in the laboratory until they were tested. A view of the walls in storage is shown in Fig. 3(a). The first two walls have already been capped. The lifting rig used in transporting the walls is shown on the floor near the walls. The temperature in the laboratory, as recorded on the autographic thermometer shown in the figure, from July 2 to September 5 ranged from 71 to 93 deg., with an average of about 83 deg. F. The relative humidity ranged from 54 to 92 per cent.

The walls were tested when the last course was 28 days old. Several days before the test the walls were capped with a layer of neat high-early-strength cement. The steel capping plate, 2 in. thick, was left in place and used as a bearing plate during the test. The walls were moved to the testing machine by means of a ten-ton traveling crane. Two ⅞-in. rods passing through pipe sleeves in the pallet provided means of attaching lifting straps suspended from the crane and also served for the purpose of attaching casters used in moving the wall into the testing machine.

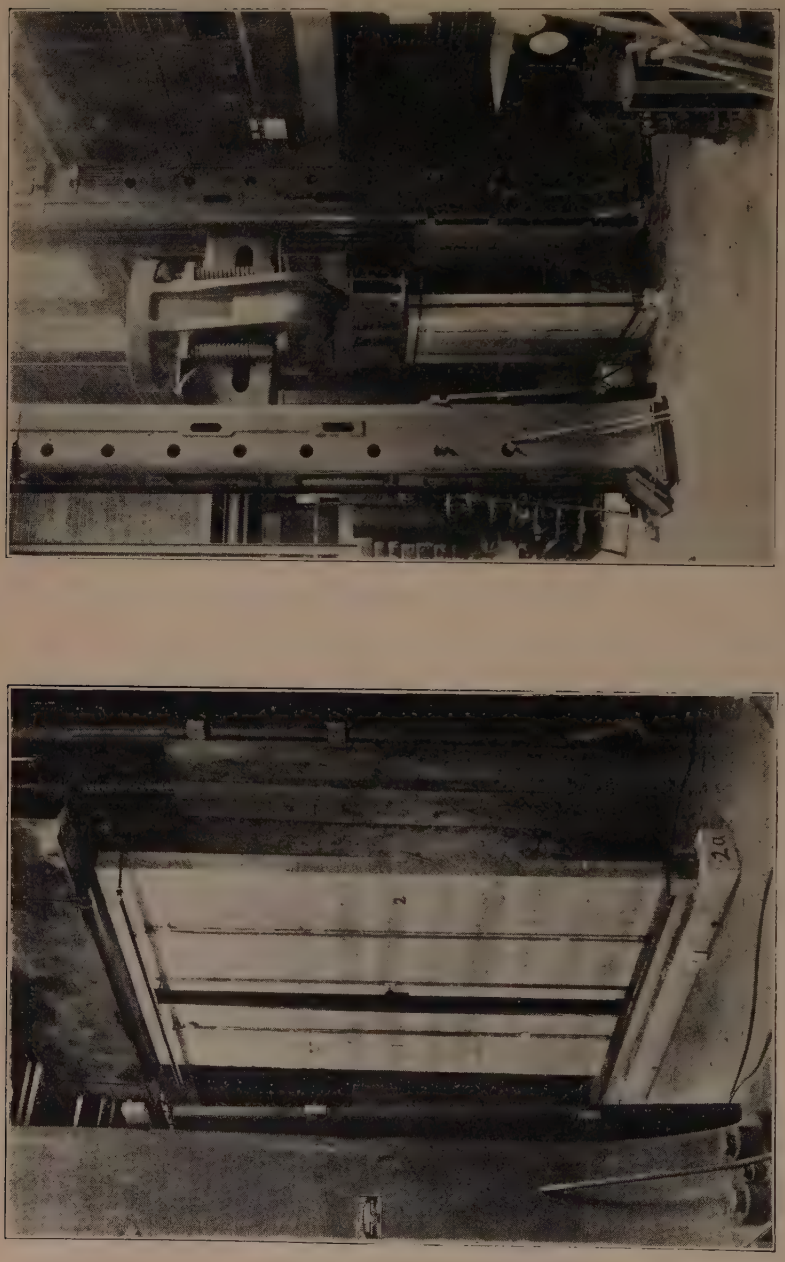
5. *Construction of Walls of Poured Concrete.*—Forms for the ribbed walls and solid poured walls were set up on pallets of the same sort as those for the walls of dry-tamped concrete. The forms consisted essentially of one-inch sheathing and two-by-four studs and wales supported on the pallet and braced by an A-frame arrangement. In general, the back of the form was built up to full height, reinforcement wired in place, and plugs set for attaching the measuring instruments to be used during the test of the walls. The front and back of the form were held together by a combination tie and spacer. The vertical bars in the ribbed wall were spaced by putting the bars through holes in the wall-strap portion of the tie. Figure 2(b) shows the form for a ribbed wall. The reinforcement and the ties may be seen in the figure. A similar type of form was used for the solid poured walls.

The concrete in the ribbed walls was placed in three batches and in the solid walls in five batches, and was carefully and thoroughly spaded as it was placed. Also, the front of the form was built up as concreting proceeded. In order to work the concrete around the mesh in the two-inch slab of the ribbed walls, it was necessary to limit the maximum size of the gravel, and to use concrete of a rather wet consistency.

The wall forms were stripped after 24 hours had elapsed, the holes left by the removable cones of the ties were pointed with mortar, and the wall was allowed to stand in the laboratory until tested. The details of capping and transporting these walls were the same as for the walls of dry-tamped concrete. The walls were tested when 28 days old. A view of the walls in storage is shown in Fig. 3(b).

Four control cylinders were made with each batch of concrete for the ribbed walls and with each alternate batch for the solid poured walls. One-fourth of the cylinders were cured in moist storage; the others were cured in the laboratory with the walls. One of each group of air-cured cylinders was tested at 7 days, the other two at 28 days. All the moist-cured cylinders were tested at 28 days.

6. *Compressive Tests of Solid and Hollow Walls.*—Compressive tests on all the solid and hollow walls were conducted in the 3 000 000-lb. Southwark-Emery hydraulic testing machine. The loading was applied axially on the solid walls and on the inside wall of the hollow walls through a spherical bearing block and two 24-in. I beams to the 2-in steel plate used to cap the wall. The I beams were sufficiently stiff to distribute the load very uniformly. Vertical de-



(a) (b)

FIG. 4. WALLS IN TESTING MACHINE

formations of the wall were measured by means of four attached vertical Ames dial compressometers of 96-in. gage length, two on each side of the wall. Lateral deflection at mid-height of the wall on a vertical center line was observed by means of an attached Ames dial deflectometer on an 8-ft. span. Load was applied in about eight or ten increments to failure, and observations were taken at each load increment. The wall was carefully centered in the machine, and the deformations for the first two load increments were measured to determine whether or not the wall was deforming uniformly. When one side showed much more deformation than the other, indicating an eccentricity in load, the wall was moved and recentered. After the second load increment had been applied, the spherical bearing block of the testing machine was wedged in position. For the first few walls tested, walls 1A, 1B, and 2A, the spherical block was not wedged. Apparently this made little difference, since wall 1B carried a higher load in relation to its concrete strength than any other 4-in. wall.

Wallettes were tested in a manner similar to that in which the large walls were tested. However, vertical deformations were measured by means of two compressometers of 36-in. gage length, one on each side of the wall. No deflection measurements were taken.

Views of walls 2A and 4A in the testing machine with instruments attached ready for test are shown in Fig. 4. The plugs to which the compressometers were attached were ½-in. square bars. The conical points of the compressometer fitted into No. 54 drill holes in the plugs. Small plugs were set in the wall to receive the points of the deflectometer which was attached by long clamps, as shown in the figure. The upper plug had a drilled hole and the lower a slot to permit movement as the wall deformed.

7. *Compressive Tests of Ribbed Walls.*—The ribbed walls were tested in somewhat the same manner as the solid and double walls. The load was applied through a spherical bearing block and two 24-in. I beams to a 2-in. steel plate, then through a ½-in. square cold-rolled steel bar to the 2-in. steel plate used to cap the wall. The ½-in. steel bar was used to center the loading over the point desired. Two tests were made with the steel bar placed over the centroid of the wall section, which gave axial compression in the wall, and two tests were made with the steel bar centered over the center of the rib, which gave a very highly eccentric loading. The spherical bearing block was in each case wedged after the second load increment was applied.

(a)

(b)

FIG. 5. ARRANGEMENT FOR TRANSVERSE TEST OF WALLS
(a) WALL IN TESTING FRAME
(b) DETAIL OF PIVOTS AND SCREW-JACK LOADING BLOCK

8. *Tests of Walls in Flexure.*—In the flexure tests the wall was supported laterally by means of horizontal steel beams 8 feet apart near the top and bottom edges of the wall attached to a steel frame, and the load was applied uniformly along the horizontal center line of the wall by means of a right-angled lever supported from the frame. The lever was loaded by means of weights hung at the end. In order to balance the dead weight of the lever, an upward force was applied at its center of gravity by means of a counter-weight hung over a ball-bearing pulley. All loading beams bearing against the wall were cushioned with a 3-in. strip of leather belting. Figure 5(a) shows a wall set up for a flexure test. The A-frame in the center of the figure supports the dead weight of the lever by means of the cable and counterweight. In Fig. 5(a) load is transmitted to the wall through a knife edge and a screw jack in order to be able to adjust the inclination of the lever as the wall deflected. A closer view of this arrangement is shown in Fig. 5(b).

For the flexure tests of the double wall the load was applied to the outside wall and the reactions to the inside wall. At the top and bottom of the panel the air space between the two walls was filled with a narrow strip of concrete to transfer the reactions from the outer wall to the inner wall. Some part of the applied load was evidently transmitted from outer to inner wall by the ¼-in. ties. The ribbed wall 8T was tested with the ribs in tension. Since walls 8A and 8B, tested under eccentric loading, failed only locally at the top of the wall, they were retested in flexure; wall 8AA (retest of 8A) was tested with the slab in tension, and 8BB (retest of 8B) was tested with the rib in tension.

For all the walls tested in flexure, except walls 8AA and 8BB, weights were placed on top of the wall to simulate the effect of dead load on a bearing wall of a house. A vertical load of approximately 300 lb. per lineal foot was carried by the walls during the flexure tests.

Except for wall 1T, deflections at the center were measured on a span between supports of about 90 inches by an Ames dial deflectometer clamped to one edge of the wall.

III. Results of Tests

9. *Principal Results of Tests.*—The principal results of the compression tests on solid and hollow walls are given in Table 3. The maximum stress carried by a wall is computed on the assumption that the maximum load is distributed uniformly over the loaded wall. It

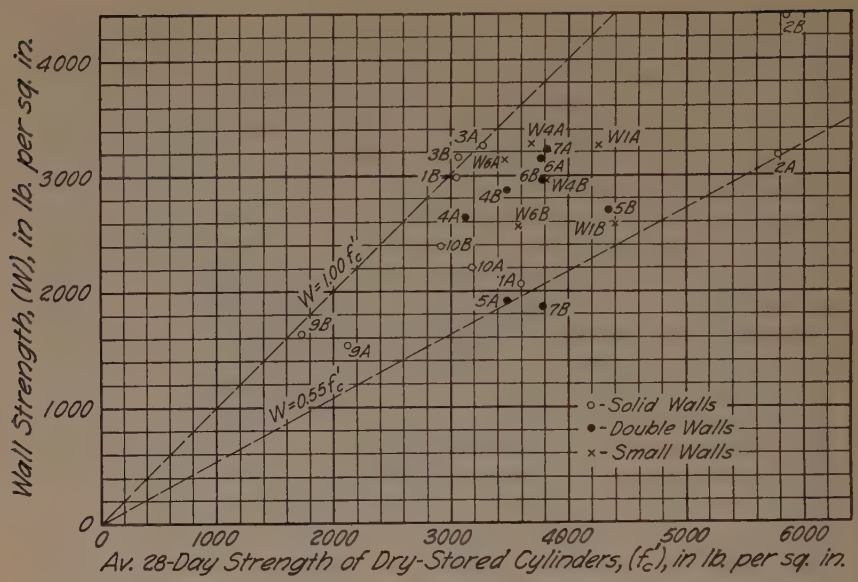


Fig. 6. Relation Between Wall Strength and Concrete Strength

should be noted that wall 7B was damaged during construction, and had to be patched. It failed in the course that was originally damaged.

The principal results of the tests on ribbed walls are given in Table 4. This table includes the tests in compression, both concentric and eccentric, and the tests of the walls in flexure. Walls 8A and 8B failed only locally at the top of the wall when eccentrically loaded. These walls were retested in flexure, and are referred to for the flexure tests as walls 8AA and 8BB, respectively.

The principal results of the tests on solid and hollow walls in flexure are given in Table 5.

In Tables 3 and 4 the average values of control cylinder strength for different conditions of storage are reported as data possessing some potential value in indicating the effect of kind of curing on the strength of a wall, and as possibly indicating the relative strength of the wall at early ages. It is of interest to note the range in strength for the different walls made of the same mix of dry-tamped concrete. A number of elements contribute to the great variability of dry-tamped concrete. Probably the most important is the degree of tamping. For walls 1A, 1B, 2A, 2B, 3A, 3B, 4A, 4B, and 5A a wooden tamp was used for the cylinders, and for the other walls a

metal tamp of approximately the same size and weight was used. The increase in cylinder strength for the remaining walls might lead one to believe the difference was due to the change in tamp. However, it should be noted that the ratios of wall strength to cylinder strength for the double 4-in. walls and for the 6-in. and 4-in. double walls are approximately the same.

10. *Effect of Strength of Concrete on Wall Strength.*—The relation between the wall strength and the average cylinder strength is shown in Fig. 6. From a study of this figure and of Table 3 it may be seen that there is a general but not very clearly defined relation between wall strength and cylinder strength which is apparently not greatly affected by the size or type of wall. Except for wall 7B which failed at an evident flaw, the lowest value of the ratio between wall strength and cylinder strength was 0.55. The average value of the ratio was about 0.78. It may be noted that in previous tests on concrete masonry walls,* 8 in. or more in thickness and of similar height and length, the wall strengths averaged from 53 to 55 per cent of the strength of the units used in the wall. The strength of even the 4-in. walls which were the most slender monolithic walls of the present series was considerably greater than that indicated for the concrete masonry walls.

From the results of tests of ribbed walls given in Table 4, it may be seen that these walls developed about 58 per cent of the strength of the material.

11. *Relation Between Compressive Strengths of Different Types of Walls.*—As far as resistance to axial compression is concerned, the different types of walls tested showed about the same efficiency. Considering the concrete strengths, the 6-in. solid walls were the strongest and developed approximately the same unit stress in the wall as the 6 by 12-in. control cylinders of the same concrete. The 4-in. solid walls carried less unit stress than the 6-in. walls. This would be expected because of their greater slenderness. The double walls were not as strong as the 6-in. solid walls. This may be explained by the manner of placing of the ties which contributed somewhat to the weakness of these walls in that the concrete around the ties was not tamped so well nor restrained so well during tamping as the rest of the concrete. The double 3-in. walls were definitely weaker than the rest, probably because of the slenderness of the 3-in. loaded wall, and in addition the weakening effect of the ties which offset, in some measure, the

*See footnote, page 5.

supporting effect of the unloaded wall tied to the loaded wall. The ribbed walls developed about 57 to 60 per cent of the strength of the concrete and steel. This proportion was not as large as in some of the other types of wall tested.

From Fig. 6 it appears that there is no very clearly defined reduction in strength of large walls compared to small walls. If any conclusion might be drawn from the small number of tests it would be that for walls of the thickness tested (that is, for walls more than 4 in. thick) the strength of small walls is representative of the strength of large walls.

None of the axially-loaded walls showed signs of distress until the load had just about reached its maximum value, and in most cases the failure of the wall was sudden and without any warning. Practically all of the solid dry-tamped walls collapsed completely from the shock of the failure. In general, most of the individual courses remained intact, and it appeared almost as if the wall had been built up of separate blocks which came apart, some of which broke when they struck the bed of the testing machine in falling. Failure generally occurred in one or two courses by shearing and crushing of the concrete. The particular course in which failure occurred could not, however, always be located. Some of the walls failed by crushing at one point from which a crack extended vertically down through several courses. An example is shown in Fig. 7.

All double walls remained standing after failure, the unloaded wall being sufficiently strong to bear the shock and to hold up the weight of the broken wall. Figure 7 shows typical examples of the manner of failure of double walls.

The same general statements are true of the manner of failure of the small walls, or wallettes, as for the large walls. A view of a small double wall after failure is given in Fig. 7 also.

The concentrically-loaded ribbed walls gave some signs of impending failure at loads very near the maximum load. However, this was probably due to slight unavoidable eccentricities in loading. The eccentrically-loaded walls showed signs of failure, such as crushing of the concrete and cracking on the tension side, at loads 5,000 lb. or more below the ultimate load. Both eccentrically-loaded walls failed by crushing of the concrete in the ribs accompanied by a slight cracking of the concrete in the slab to a depth of about an inch. The walls failed at the top almost directly under the spandrel beam.

12. *Deformations of Solid and Hollow Walls.*—Stress–strain curves for all solid and hollow axially-loaded walls are given in Figs. 8 and

FIG. 7. VIEWS OF WALLS AFTER FAILURE

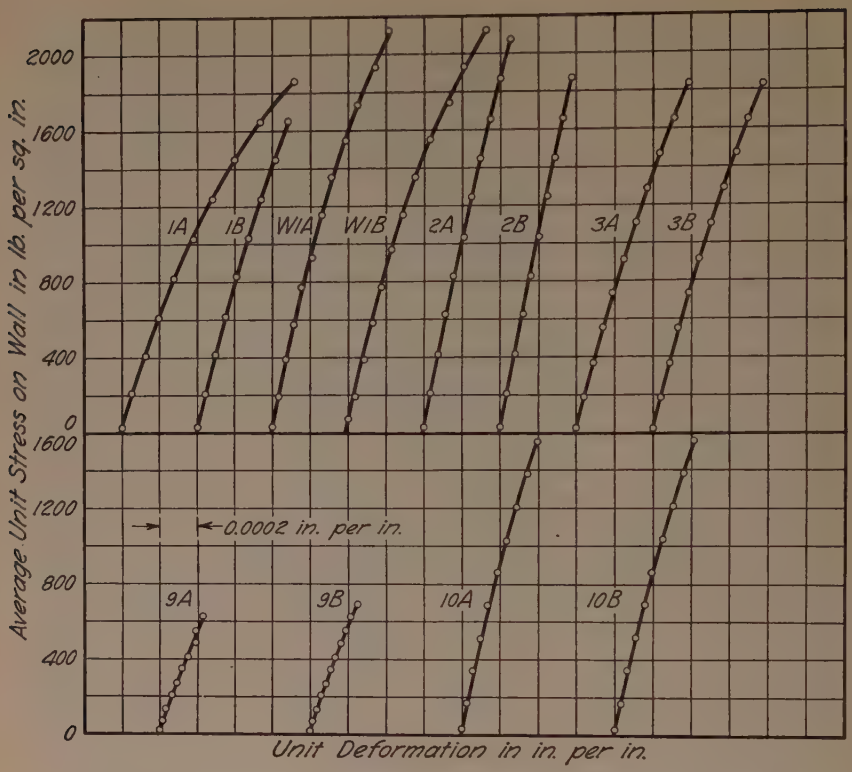


FIG. 8. STRESS–STRAIN CURVES FOR AXIALLY-LOADED SOLID WALLS

9. For the large solid walls each curve represents the average of four strain measurements on 96-in. gage lines, and for the small solid walls, the average of two measurements on 36-in. gage lines. For the large double walls each curve represents the average of two strain measurements on 96-in. gage lines on the same side of the wall, and for the small double walls, a single measurement on a 36-in. gage line. Curves are drawn for both the loaded and unloaded wall of the double wall.

The slope of the lower straight portion of the stress–strain curve (for the loaded walls) is the tangent modulus of elasticity of the wall. Values of the moduli, taken from the curves, are tabulated in Table 6. The values are fairly high, and there is a tendency for the higher values to correspond to the higher concrete strengths. Under working loads the stress–strain curve is practically a straight line. The elastic deformations under working loads are very small.

The deflections of the walls during the test were measured. How-

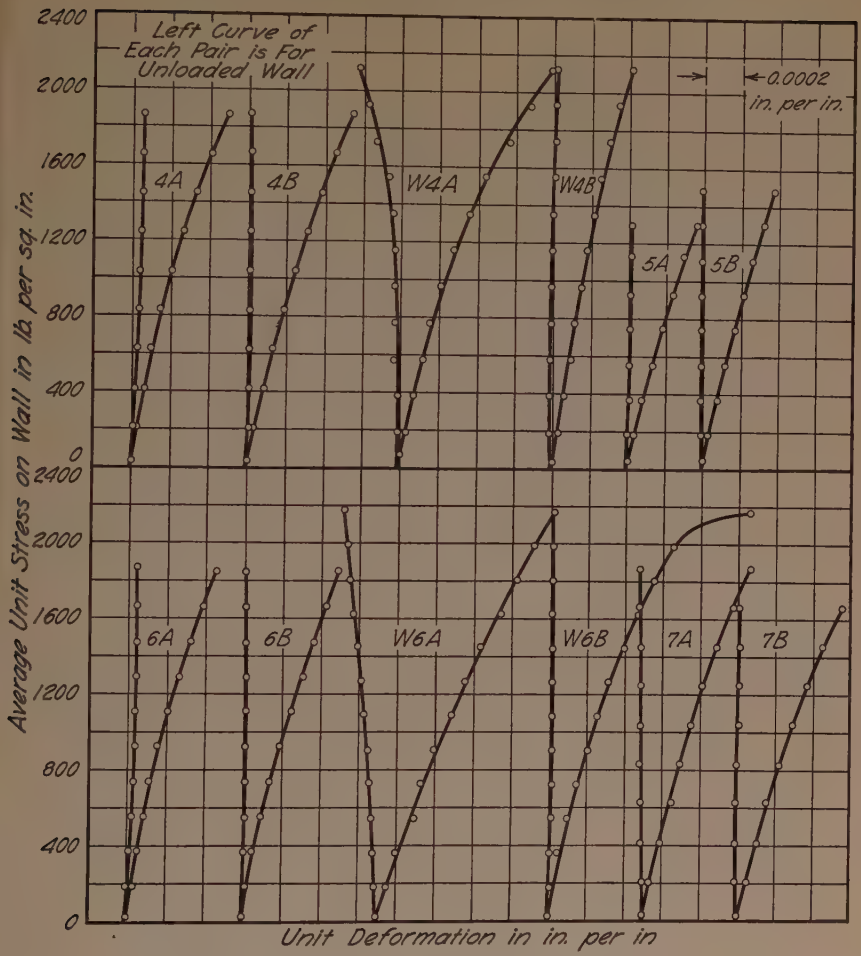


FIG. 9. STRESS–STRAIN CURVES FOR AXIALLY-LOADED DOUBLE WALLS

ever, the measuring instruments were removed at about half the ultimate load. The recorded deflections obeyed no general rule. In a number of walls the deflection reached a maximum and then dropped off with increasing load. Of the maximum deflections recorded, values for only three walls were greater than $\frac{1}{16}$ in., and the maximum value was 0.095 in. for wall 3A. This wall, incidentally, developed over 99 per cent of the cylinder strength.

13. *Strength of Ribbed Walls in Compression.*—Two ribbed walls were tested with a uniformly-distributed load applied as nearly as

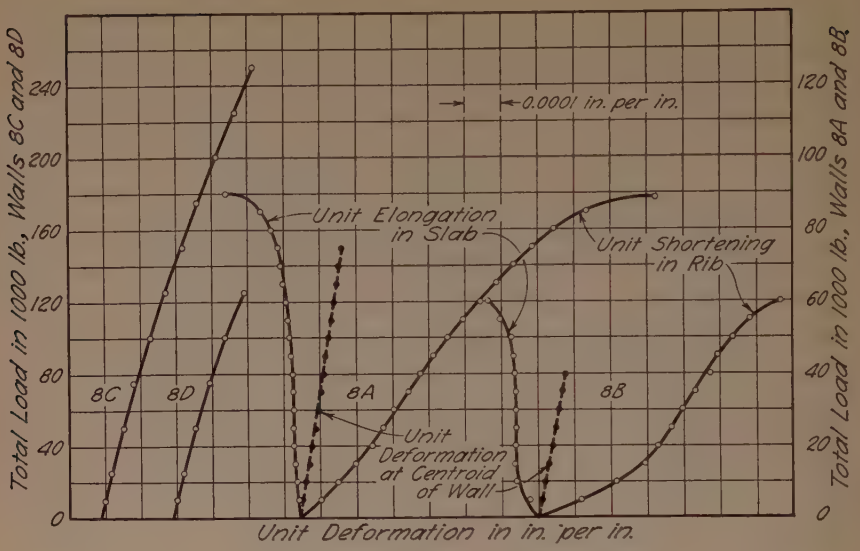

Total Load in 1000 lb., Walls 8C and 8D
Total Load in 1000 lb., Walls 8A and 8B
0.0001 in. per in.
Unit Elongation in Slab
Unit Shortening in Rib
Unit Deformation at Centroid of Wall
8C
8D
8A
8B
Unit Deformation in in. per in.

FIG. 10. LOAD–STRAIN CURVES FOR RIBBED WALLS

possible along the centroidal axis of the wall. In computing the position of the centroid the value of n, the ratio of the moduli of elasticity of steel and concrete, was assumed to be 10, and for this value of n the centroid is 1.54 in. from the face of the slab. It should be pointed out that for $n = 0$, the centroid is located 1.51 in. from the face of the slab, and for $n = 20$, 1.57 in. from the slab face, a difference of less than $\frac{1}{16}$ in. Each of the two walls carried a maximum load of approximately 300 000 lb. Computing the concrete stress on the basis of $n = 10$, the walls developed 60.6 and 64.3 per cent of the concrete strength. Considering the possible wall strength to be the sum of the strengths of its parts (that is, the concrete area to develop the concrete cylinder strength and the steel area to develop the yield point stress) the walls developed about 58 per cent of the possible strength.

The two walls tested with a load applied over the center of the rib, that is, 2 in. from the back of the rib, carried 63 300 and 88 700 lb., about 20 to 30 per cent of the load carried by the concentrically-loaded walls. Using the position of the centroid of the uncracked section as computed, the eccentricity of a load applied at the center of the rib is 2.46 in. When the slab begins to crack, the eccentricity changes. Although visible cracks were not noticed until the ultimate load was approached, apparently there was some slight cracking, as was shown by the measurements of deformation.

The load–strain curves for the ribbed walls are given in Fig. 10. The curves for the concentrically-loaded walls represent the average of strains on four 96-in. gage lines. The curves for each side of the eccentrically-loaded walls represent the average of strains on two 96-in. gage lines. It should be pointed out that strains were measured about an inch in front of the wall face. From the values so obtained the strains at the wall face were computed, assuming a linear distribution of strain across the section, and the values computed were used in drawing the stress–strain curves. A curve is also shown for the eccentrically-loaded walls giving the computed strains at the centroid of the wall. The curve is drawn only as far as it remained fairly straight. For higher loads, when cracking occurred, the centroid shifted, and the strains at the original centroid increased less rapidly.

Values of the modulus of elasticity obtained from these curves are reported in Table 6. The results from the eccentrically-loaded walls are considerably lower than from the axially-loaded walls, and are probably in error because of less precision in the values computed on the various assumptions from the measured deformations.

Deflections of the walls were measured during the tests, and for the concentrically-loaded walls the average value was about 0.03 in. when the instruments were removed. For the eccentrically-loaded walls the deflections were fairly large; approximately 0.13 in. for wall 8A at 88 700 lb., and 0.14 in. for wall 8B at 60 000 lb. The load–deflection curves (not shown) were fairly straight at all loads.

It was the object of the eccentric load tests to simulate the effect of loading the wall through joists bearing on the ribs. It may be pointed out that a larger width of rib would add considerably to the strength of a wall loaded in this manner, due both to the increase in section and to the decrease in eccentricity of a load applied to the center of the rib. Another means of strengthening the wall would be to eliminate bearing on the outer portion of the rib, by means of chamfering the edge or inserting a narrow layer of compressible material.

14. *Flexural Strength of Walls.*—A summary of the flexural tests of the solid and hollow walls is given in Table 5. All the dry-tamped walls failed very suddenly and without any particular warning. The manner of loading, through a lever, was such that any large motion of the lever could be arrested and the horizontal thrust on the wall released by allowing the lever to come to rest on a supporting block when the wall deformed any considerable amount. In all cases the

TABLE 3

PRINCIPAL RESULTS OF COMPRESSION TESTS OF SOLID AND HOLLOW WALLS

All walls made of dry-tamped concrete except No. 10.
Maximum stress is maximum load divided by average cross-sectional area of loaded wall. For double walls, the load was applied to the inside wall.

Group and Wall No.	Description of Wall	Maximum Stress lb. per sq. in.	Control Cylinder Strength*			Ratio, Wall Strength to Average 28-day Dry-stored Cylinder Strength
			Average 7-day Dry-stored lb. per sq. in.	Average 28-day Moist-stored lb. per sq. in.	Average 28-day Dry-stored lb. per sq. in.	
1A	Large solid 4-in. wall.....	2070	2910	3950	3600	57.4
1B		2980	2420	3150	3050	97.7
2A	Large solid 4-in. wall.....	3180	4690	5700	5790	55.0
2B		4380	5140	6020	5850	74.9
3A	Large solid 6-in. wall.....	3260	2670	3120	3280	99.4
3B		3180	2490	3190	3070	103.5
4A	Large double 4-in. wall.....	2640	2490	3220	3120	84.5
4B		2880	2800	3610	3480	82.7
5A	Large double 3-in. wall.....	1910	2800	3610	3480	54.9
5B		2690	3190	4330	4340	62.1
6A	Large double wall, 6 in. and 4 in.	3160	2990	4070	3760	84.0
6B		2960	2830	4210	3790	78.2
7A	Large double wall, 4 in. and 3 in.	3220	2990	4180	3810	84.6
7B		1870†	2830	4210	3790	49.2†
W1A	Small solid 4-in. wall.....	3260	3140	4470	4260	76.4
W1B		2590	3230	4220	4400	58.8
W4A	Small double 4-in. wall.....	3280	2980	3920	3680	89.2
W4B		2950	3010	4220	3840	76.8
W6A	Small double wall, 6 in. and 4 in.	3140	2490	3450	3460	0.8
W6B		2560	2800	4280	3570	71.8
9A	Large solid 4-in. wall.....	1530	1530	2500	2120	72.2
9B		1610	1360	2080	1730	93.2
10A	Large solid 4-in. wall.....	2200	2150	3600	3190	69.0
10B		2400	1990	3530	2910	82.3

*Values given are the average of tests on 6 to 12 cylinders for the large walls and 4 cylinders for the small walls.
†Failed in course damaged by tamping during construction.

weight on the lever was sustained without any appreciable creep until the maximum load was reached, when the lever came down to rest suddenly on the blocking. The wall at this time was cracked and did not carry any further load. The solid-poured wall gave a little more notice of impending failure by slight creeping before cracking occurred, but this creep was so small as to be almost negligible, and when failure occurred the lever came down upon the block very suddenly. No cracks were noticed before failure occurred.

The tamped walls all failed at a daily construction joint at a modulus of rupture of 60 to 120 lb. per sq. in. The modulus of

rupture for the tamped walls was not very much higher than the modulus of rupture for masonry walls,* and apparently does not depend on the flexural strength of the concrete, but rather on the adhesion between adjacent courses. The solid wall made of poured concrete failed at a modulus of rupture of about 400 lb. per sq. in. This indicates the possibility of increasing the strength in flexure of dry-tamped walls either by the use of vertical reinforcement, or by constructing the joint in a different way, to develop better bonding of adjacent courses.

The flexural tests of the ribbed walls are summarized in Table 4. The ribbed walls were all reinforced and, of course, the phenomena of failure of the transversely-loaded ribbed walls were entirely different from those of the solid walls which were without vertical reinforcement. A sudden jump or change in deflection was noted when the first cracks were visible in the tension side of the wall. After cracking occurred, a slight creep was noticed on the dial used for measuring deflections when the load was left on for a short length of time such as, for instance, when the weights on the scale pan were changed. The two walls tested with the rib in tension, walls 8T and 8BB, showed increasing amounts of creep as the loads grew larger and it required more or less of an approximation to determine the ultimate load. Initial failure of the walls occurred by yielding in the reinforcing steel. The deflection, as the load neared its ultimate value, was of the order of ½ in., and when further load was applied the lever gradually swung down toward the support as the wall yielded. The wall probably could not have withstood these loads for any great length of time without complete failure. In the case of wall 8AA, tested with the slab in tension, the phenomena were slightly different. There was a small amount of creep as loads were applied, but a definite ultimate load was reached when the maximum load was applied, and the wall failed by compression in the rib. The lever came down to rest rapidly and the wall was unable to support the maximum load again, even momentarily.

The ribbed walls carried very high transverse loads before failure occurred. The arrangement of reinforcement in the section is different from that in the ordinary T beam, and consequently the interpretation of the test results is affected. Upon computing the nominal steel stresses in the rib reinforcement for walls 8T and 8BB, tested with the slab in compression, the values are found to be well above the ultimate strength of the steel. However, when it is recognized that

*"Strength and Stability of Concrete Masonry Walls," Univ. of Ill. Eng. Expt. Sta. Bul. 251, 1932.

TABLE 4

PRINCIPAL RESULTS OF TESTS OF RIBBED WALLS

Wall No.	Kind of Test	Age at Test, days	Manner of Failure	Maximum Vertical Load, P lb.	Eccentricity* in.	Maximum Moment†		Computed f_c‡	Control Cylinder Strength¶ lb. per sq. in.			F§	$\frac{P}{F}$ per cent
						Total in. lb.	Per ft. in. lb.		Av. 7-day Dry-stored	Av. 28-day Moist-stored	Av. 28-day Dry-stored		
8A	Eccentric compression	28	Compression in rib	88 700	2.46	.....	.....	3620	1750	3330	2610	476 000	18.6
8B	Eccentric compression	28	Compression in rib	63 300	2.46	.....	.....	2700	2240	3910	3460	611 000	10.3
8C	Axial compression	28	Compression	319 000	0	.....	.....	1900	1970	3830	3130	559 000	57.2
8D	Axial compression	27	Compression	294 000	0	.....	.....	1750	1840	3220	2720	494 000	59.6
8T	Flexure	28	Tension in steel	1 530	....	−165 100	30 100	....	1850	3300	2720	.....	....
8BB	Retest of 8B in Flexure	38	Tension in steel	0	....	−130 100	23 600	....	....	....	....	.....	....
8AA	Retest of 8A in Flexure	45	Compression in rib	0	....	+196 500	35 600	....	....	....	....	.....	....

*Centroid of transformed section computed on assumption $n = 10$.
†Moment at center of 8 ft. span due to single concentrated horizontal load. Plus (+) moment signifies compression in rib.
‡Stress computed assuming straight line distribution, $n = 10$.
¶Values for 7-day dry-stored and 28-day moist-stored cylinders are average of 3 tests. Values for 28-day dry-stored cylinders are average of 6 tests.
§$F = A_c f_c' + A_s f_s^{Y.P.}$ where A_c = Area of concrete = 159.1 sq. in.
f_c' = Av. 28-day strength, dry-stored cylinders, lb. per sq. in.
A_s = Area of steel, 0.463 sq. in. of mesh, 0.393 sq. in. for rib steel.
$f_s^{Y.P.}$ = Yield point of steel; rib steel, yield point = 51 500 lb. per sq. in., ultimate strength = 77 900 lb. per sq. in.; mesh (cold drawn wire), no yield point—control specimens broke in tension in welds at 87 000 lb. per sq. in. (yield strength at 0.002 permanent set approximately 87 000 lb. per sq. in.)

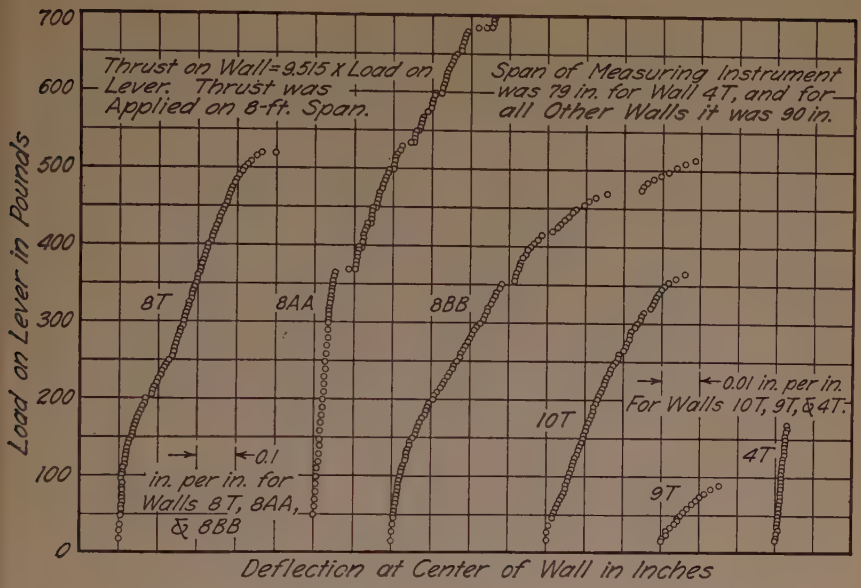

Thrust on Wall = 9.515 X Load on Lever. Thrust was Applied on 8-ft. Span.
Span of Measuring Instrument was 79 in. for Wall 4T, and for all Other Walls it was 90 in.
Load on Lever in Pounds
700
600
500
400
300
200
100
0
8T
8AA
8BB
10T
9T
4T
0.1 in. per in. for Walls 8T, 8AA, & 8BB
0.01 in. per in. For Walls 10T, 9T, & 4T.
Deflection at Center of Wall in Inches

FIG. 11. LOAD–DEFLECTION CURVES FOR WALLS IN FLEXURE

the wire mesh at the center of the T flange was also in tension, that the rib steel had a very definite yield point of about 50 000 lb. per sq. in., and that the wire mesh of cold-drawn wire had no definite yield point, then it is apparent that the stress in the mesh might well have been greater than the stress in the rib steel at failure even though the mesh was much closer to the neutral axis. It is interesting to note that the ribbed wall tested with the ribs in compression was stronger than the walls tested as T beams with the slab in compression. This is due to the arrangement and amount of reinforcement. It should be noted that in the case of the walls where the critical moment occurred at the center of the wall, the moments given in Table 4 are slightly too high because the load is assumed to be concentrated. Actually the load was applied through a 3-in. strip of leather belting. Computing moments for this loading would result in values about 1½ per cent less than those given. There is a further possibility of reduction in moment due to bearing of the wall on the edges of the support and loading beams when the deformations are high. This could cause a total reduction of about 7 per cent in the center moment.

The deformations of the ribbed walls during the test are plotted in Fig. 11. It is of interest to note the stiffness of wall 10T compared to walls 8T and 8BB.

Table 5

Principal Results of Flexural Tests of Solid and Hollow Walls

Single concentrated load applied at center of 8-ft. span
All walls made of dry-tamped concrete except 10T.
Top of course 6 at center of wall.

Wall No.	Description	At Center of 8-ft. Span			Point of Failure	At Section Where Failure Occurred				Av. 28-day Strength, Dry-stored Cylinders¶
		Maximum Moment		Nominal Flex. Stress		Moment, per ft.	Flex. Stress	Compression†	Resultant Modulus of Rupture	
		Total in. lb.	Per ft.* in. lb.	lb. per sq. in.		in. lb.	lb. per sq. in.	lb. per sq. in.		lb. per sq. in.
1T	Single 4-in. wall.....	16 640	2760	86	Top of course 5	2240	70	7	63	3350
4T	Double 4-in. wall....	65 000	5380‡	169‡	Top of course 5	4370‡	137‡	7	130‡	4340
9T	Single 4-in. wall.....	20 800	3440	108	Top of course 4	2150	67	8	59	2120
10T	Single 4-in. wall.....	79 900	13200	413	Center	13200	413	6	407	3160

*Width of wall 4T, 72.2 in.; width of other walls, 72.5 in.

†Compressive stress due to vertical load on wall of 315 lb. per ft. for walls 1T, 4T, 9T, and 250 lb. per ft. for wall 10T, plus weight of wall above section where failure occurred.

‡Each part of double wall assumed to take one-half the thrust.

¶Values are average of 6 to 10 cylinders.

The relative strengths of the various walls in flexure may probably be best compared by considering the moment carried per foot of width of wall. Considering the walls built of dry-tamped concrete, the single 4-in. walls carried about 2200 in. lb. per ft., and the double 4-in. wall carried about 4400 in. lb. per ft. per wall, or 8800 in. lb. per ft. of double wall. Of the poured walls, the solid wall carried 13 200 in. lb. per ft. of wall, and the ribbed walls carried 23 600 to 30 000 in. lb. per ft. of wall. The ribbed walls were the only walls which had vertical reinforcement. It may be well to compare with these figures the moment produced on a 9-ft. story-height span by a uniformly applied load, as, for example, a wind load. A load of 20 lb. per sq. ft. produces a maximum moment of 2430 in. lb. per ft., and a load of 30 lb. per sq. ft. a moment of 3650 in. lb. per ft.

15. *Significance of Results.*—The foregoing tests have dealt only with elements relating to the structural requirements of monolithic concrete walls, such as resistance to vertical and lateral loads. The significance of other properties, including fire resistance, insulating qualities, and durability is not to be minimized because they have not been investigated here; however, it seems logical that matters of strength and stability should be given first consideration when the thickness of a concrete bearing wall for a residence is to be determined. Although vertical loads carried by a wall are generally specified as axial and uniformly-distributed, such loads may be concentrated and are quite often eccentrically applied. Eccentricity is produced by bearing of the floor or roof members on the inner edge of the wall due either to the design of the bearing surfaces or to the deflection of the structural members under load. The effect of eccentric loading should be borne in mind. The ribbed walls loaded at the center of the rib were able to carry only about 20 to 30 per cent as great a load as the walls under an axial load. It may be pointed out that most of the eccentric loads encountered may be avoided by proper design and detailing.

Lateral forces on a wall are those due to wind, earthquakes, and, occasionally, unbalanced roof thrust. Factors which are of prime importance in determining the resistance of a wall to the vertical and lateral forces enumerated are the effect of openings and of slenderness of the wall. Further, there is the effect of the vertical dead load counteracting the tensile stresses produced by lateral forces. In connection with transverse loads it may be pointed out that flexure may take place in either or both of two directions, depending upon the

TABLE 6

MODULUS OF ELASTICITY OF CONCRETE IN WALLS

Wall No.	Modulus of Elasticity $\times 10^6$ lb. per sq. in.	Concrete Cylinder Strength, Av. 28-day, Dry-stored lb. per sq. in.
1A	3.6	3600
1B	4.1	3050
W1A	5.0	4260
W1B	4.3	4400
2A	5.3	5790
2B	5.3	5850
3A	3.7	3280
3B	3.9	3070
4A	5.6	3120
4B	4.6	3480
W4A	4.7	3680
W4B	6.7	3810
5A	4.4	3480
5B	4.6	4340
6A	6.7	3760
6B	5.9	3790
W6A	3.1	3460
W6B	4.9	3570
7A	4.4	3810
7B	3.6	3790
9A	2.8	2120
9B	2.4	1730
10A	4.9	3190
10B	4.5	2910
8C	4.9	3130
8D	4.6	2720
8A	3.7	2610
8B	3.2	3460

position of the supports and the rigidity of the wall panel in various directions. Walls of dry-tamped concrete have much more flexural resistance when spanning in the horizontal direction than in the vertical direction.

When the working stress is given in terms of the ultimate concrete strength, the minimum factor of safety in uniformly-applied axial compression for the walls tested may be computed from the fact that all walls developed at least 55 per cent of the ultimate cement strength. For a working stress of $0.10\ f'$, the factor of safety is at least 5.5, and for a working stress of $0.15\ f'$, the factor of safety is at least 3.6. For the 6-in. walls, these factors of safety are at least 30 per cent higher.

Against a uniform lateral wind load of 20 lb. per sq. ft., and considering a 9 ft. story height with only 300 lb. per ft. vertical load on the wall, the factor of safety of the 4-in. solid dry-tamped walls is less than 1.0 unless the walls are supported along the sides. For the double wall under similar conditions, the factor of safety is about 3.6; for the poured wall, 5.4; and for the ribbed walls, at least 9.5.

From the supplementary data obtained from the control cylinders

it appears that the gain in strength from 7 to 28 days in air storage is about 500 to 1000 lb. per sq. in. for the dry-tamped mixes. The 28-day strength of moist-stored cylinders is but slightly greater than that of air-stored cylinders. This possibly may be accounted for by the relatively high humidity of the laboratory during the time the dry-tamped concrete was curing.

Particularly concerning the dry-tamped walls, the effect of constructing the walls in the laboratory instead of in the open air may be considerable. Exposure to direct sunlight and wind may cause a quicker drying and hardening of the wall concrete and permit placing successive courses at shorter intervals than was found practicable in the laboratory.

IV. Conclusions

16. *Summary of Conclusions.*—The results of the tests may be summarized as follows:

(1) The compressive strength of all axially-loaded walls varied from 1530 to 4380 lb. per sq. in. of loaded wall. The wall strength was affected to a slight extent by the type of wall, whether single, double, or ribbed, and to a very great extent by the strength of the concrete composing the wall.

(2) The compressive strength of all axially-loaded walls, large and small, was over 55 per cent of the strength of the concrete control cylinders, and averaged about 78 per cent of the cylinder strength. Only the 6-in. single walls had a strength equal to the control cylinder strength, although two 4-in. walls approached the cylinder strength very closely.

(3) None of the double walls was as strong relatively as the thickest solid walls, considering that only one of the double walls was loaded. The strongest double walls averaged about 82 to 84 per cent of the cylinder strength. There was apparently some weakening effect due to the manner of placing of the ties. The double 3-in. walls were distinctly the weakest walls tested, in comparison to the cylinder strengths.

(4) From the small number of tests no definite conclusion as to the relative strength of large and small walls may be drawn. Apparently the large walls developed about the same relative strength as the small walls of the same thickness.

(5) The modulus of elasticity of the concrete in the walls was in general about the same as has been observed previously for concrete of the same strength. The modulus varied from 2 400 000 lb.

per sq. in. to about 6 000 000 lb. per sq. in. for concrete strengths of 1730 to 5850 lb. per sq. in.

(6) The flexural strength of the dry-tamped walls was evidently dependent upon the adhesion between courses, and was least where the courses were constructed 12 hours or more apart. Values of modulus of rupture varied from 60 to 130 lb. per sq. in. for these walls.

(7) The flexural strengths of the solid-poured wall and the ribbed walls were not sensibly different from beam strengths of beams of similar section when all the elements involved were evaluated.

(8) Assuming failure in flexure to take place at the maximum moments recorded in the test walls, and further assuming construction joints in the dry-tamped walls at mid-height, the lateral loads that would cause failure of the walls on a 9-ft. story height are as follows: for solid 4-in. walls of dry-tamped concrete, 18 lb. per sq. ft.; for double 4-in. walls of dry-tamped concrete, 72 lb. per sq. ft.; for 4-in. solid-poured walls, 108 lb. per sq. ft.; and for ribbed walls, about 200 lb. per sq. ft.

(9) The eccentrically-loaded ribbed walls were able to carry only 20 to 30 per cent as great a load as the axially-loaded ribbed walls. The eccentricity was very high, almost four-tenths of the total depth of the section. Heavier rib sections might very profitably be used in walls of this kind as well as special details for minimizing the eccentricity of loading.

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